

PREDICTION STUDY — SOFT SOIL SETTLEMENTS

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SYNOPSIS

This prediction study relates to a bulk storage facility constructed over soft, deep alluvial sediments at the mouth of the Brisbane River. The site was reclaimed from tidal mud flats by placement of dredged sand filling, which was placed to excess height to preload the underlying clays. The upper loose sands were densified by vibrocompaction and the stockpile building and reclaim tunnel constructed on the surface after removal of the preload.

Participants were asked to predict settlement, rate of settlement and stability of the loaded shed.

1. INTRODUCTION

This case history/prediction exercise was introduced by Hollingsworth Consultants Pty. Ltd., who undertook geotechnical investigation for a proposed cement manufacturing plant for Adelaide Brighton Cement on a site on Fisherman Islands at the mouth of the Brisbane River.

The clinker storage shed comprised a 150 m x 40 m steel frame structure with 5 m high concrete side walls and a central reclaim tunnel. Storage of cement clinker was proposed to a height of up to 16 m with a total shed capacity of 90,000 tonnes.

From the soil data provided, participants were asked to predict settlements in response to shed loads, both without and after ground improvement and also to assess stability and rate of improvement of shear strength under load.

2. SUBSURFACE CONDITIONS

The site lies within an area of deltaic alluvium (tidal mud flats) at the mouth of the Brisbane River, which has been reclaimed by placement of dredged sand filling. Over most of the proposed construction area, filling had been placed to a height of 3 - 5 m as a preload.

Eight bores were drilled over the site and typical results are given in Fig. 1, together with the information from a continuous cone penetration test in Fig. 2. These indicate 8 - 11 m of sand (most of it dredged filling) overlying clays which are soft near the surface and grade to stiff at depths of around 16 - 18 m. Vane shear tests taken in soft clays at a depth of about 10 m indicated shear strengths of 12 - 26 kPa.

Supplementary investigation was carried out after the preload had been in place about 1 year. Measured vane shear strengths were in the range 21 - 30 kPa with an average of about 25 kPa. A permeability test carried out in the soft clay yielded a value $k = 1 \times 10^{-7}$ cm/sec.

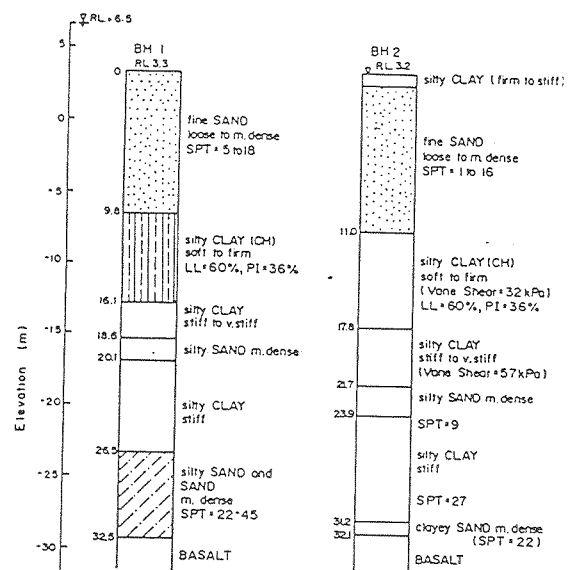


Figure 1. Typical Borehole Results

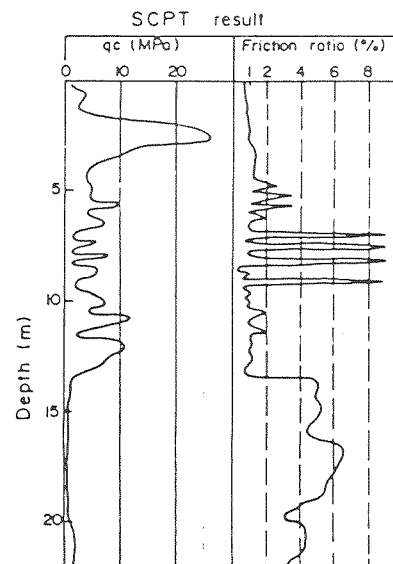


Figure 2. Typical Cone Test Results

3. LABORATORY TESTING

Laboratory testing carried out as part of the initial investigation comprised 3 oedometer tests which gave average results of:

$$m_v = 4 \times 10^{-4} \text{ kPa}^{-1}$$

$$c_v = 0.16 \text{ m}^2/\text{year}$$

Testing of a sample of clay not subject to preloading, indicated a value of $c_u = 18 \text{ kPa}$ for unconsolidated conditions and $c_u = 42 \text{ kPa}$ after consolidation for a short period under a confining pressure of 150 kPa.

4. CONSTRUCTION

The preload was placed to a height of 3 - 5 m above the final site level in March - May, 1983 and left in place until late 1984. Vibro-compaction of the upper sands was carried out in March - May, 1984.

The shed was completed and loading commenced in April, 1985. For the first twelve months, the load was limited to a maximum height of about 4 m.

Piezometers were installed in the soft clays at several locations around the site, prior to loading. These were monitored during the initial loading period.

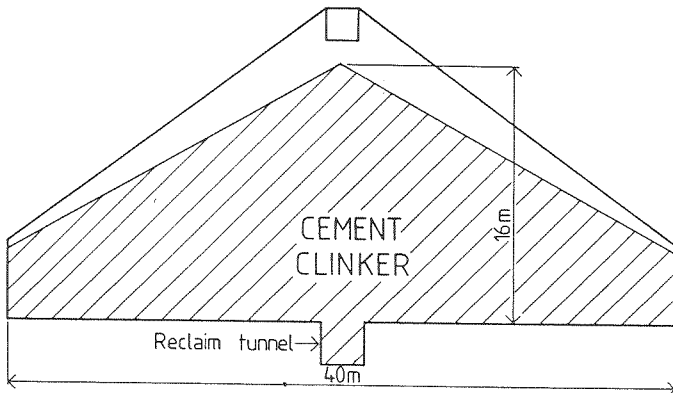


Figure 3. Cross-section of Storage Shed

5. PREDICTIONS

Participants were asked to predict the various components of settlement (immediate, consolidation and secondary) in both the upper sands and in the clays, together with the time rate of settlement in each layer. Prediction of the minimum factor of safety against rotational slip failure for the fully loaded building was also requested.

Only two predictions were received, although there had been a much greater number of requests for the data. It is suspected that both the relatively limited nature of the data and the fact that most of the "predictions" were hypothetical and could not have been measured, would have tended to discourage participation (a lesson for future prediction session organisers).

The two predictions may be compared with that of the initiators Hollingsworth Consultants (who were brave enough to document their design

estimates) and with the measured values. Only total settlement is reported here since it is the only quantity actually measured.

Participant 1 was a well seasoned campaigner, who was not deterred by the limitations of the data, and utilised experience in similar soils, particularly the nearby Brisbane Airport. He estimated very marginal stability at the commencement of loading (Factor of Safety = 1). Settlements were estimated to be about 700 mm under the centre of the stockpile and 400 mm under the edge, with additional secondary consolidation of about 100 mm/log cycle. Of the primary consolidation, it was estimated that 90% would be complete within 3 years.

Participant 2, an overseas visitor without the benefit of local knowledge, utilised the site data at face value, particularly in regard to the time rate of settlement using the measured c_v values. Total settlement was estimated at 350 mm under the centre and 170 mm under the edge, but of this only 100 mm and 50 mm, respectively, would occur within 30 years.

From the initial data, Hollingsworth initially predicted total settlement of 1300 mm under the centre and 800 mm under the edge, of which total, 70% was expected to be achieved in 50 years. This estimate was revised following completion of static cone testing after the vibrocompaction, to indicate settlements after 50 years of 450 mm under the centre and 250 mm under the edge.

The maximum actual settlement recorded in early July, 1988 was 576 mm under the centre of the tunnel and 319 mm and 409 mm under the eastern and western edges. The distributions of settlement longitudinally along the tunnel on five of the reading days (up to December 1986) are given in Fig. 4. At that time the maximum settlement was 425 mm, 80% of the latest measurements. The shape of the distribution for the settlement under the side walls shows a similar pattern to that for the centre line.

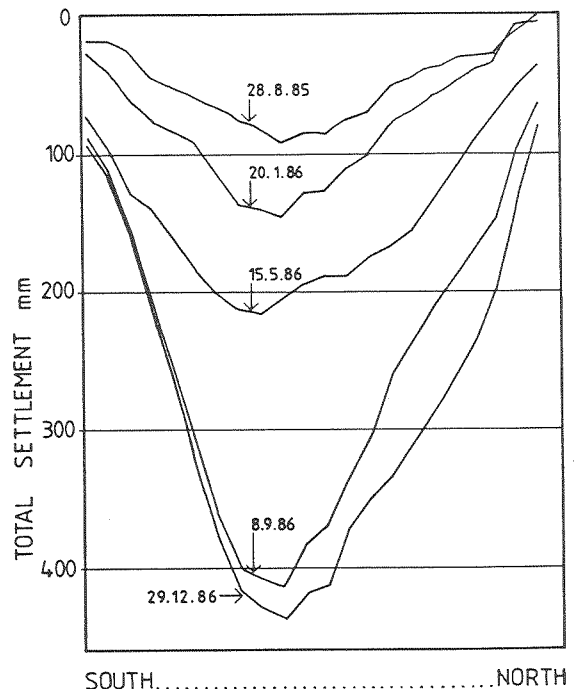


Figure 4. Settlement Profiles

A surprising feature of the settlement profiles is the variation in settlement, longitudinally along the shed. Details of loading are not available, but it is reported that the shed has been filled to capacity on several occasions. If it were uniformly loaded and ground conditions were uniform, it would be expected that settlements would be reasonable uniform over the length of the shed, except for the last 15 - 20 m at each end.

Comparison of predictions shows:

Table 1 Comparison of Predictions

	Centre (mm)	Edge (mm)
Hollingsworth (50 years)	450	250
Participant 1 (3 years) (Eventually)	630+ 700+	310 350
Participant 2 (30 years) (Eventually)	100 360	50 170
Actual (to date - still continuing)	580	410
" (maximum value)		
+ plus creep - 100 mm/log cycle		

6. CONCLUSIONS

This prediction case study tended to highlight some of the problems that occur both in predictions and in prediction exercises:

- . In a real practical problem situation the extent of data is often less than ideal.
- . Interpretation and prediction in such circumstances may need to call on other local knowledge and experience in similar situations.
- . The 2 participants made very substantially different predictions of settlement: at least part of this was due to widely varying estimates of rate of consolidation.
- . In any real situation, settlements can usually be measured reliably but it is difficult to obtain competent information regarding loading and duration.

7. ACKNOWLEDGEMENTS

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